भारतीय मानक
Indian Standard

IS 4651 (Part 3) : 2020

पत्तनों और पोताश्रयों की योजना और रूप — रीति संहिता

भाग 3 भार

(दूसरा पुनरीक्षण)

Planning and Design of Ports and Harbours — Code of Practice

Part 3 Loading

(Second Revision)

ICS 93.140

भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS
मानक भवन, 9 बहादुरशाह ज़फर मार्ग, नई दिल्ली – 110002
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI-110002
www.bis.gov.in www.standardsbis.in

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Ports, Harbours and Offshore Installations Sectional Committee, CED 47

FOREWORD

This Indian Standard (Part 3) (Second Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Ports, Harbours and Offshore Installations Sectional Committee had been approved by the Civil Engineering Division Council.

Based on the need felt towards formulating Indian standard recommendations relating to various aspects of waterfront structures, the IS 4651 series of standards were established. This standard is one of this series formulated on this subject and deals with loading. The other parts in the series are given below:

Part 1 Site investigation

Part 2 Earth pressures

Part 4 General design considerations

Part 5 Layout and functional requirements

This standard (Part 3) was first published in 1969 and subsequently revised in 1974 to provide details about ships characteristics and the methods for determining wave forces. In the first revision, due weightage was given to international coordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country.

The following significant changes have been made in this revision:

a) Information about vertical imposed loads due to container cranes and mobile harbour cranes have been added.
b) Details regarding berthing energy has been modified.
c) The calculation of wave force has been modified to determine the total force acting on the vertical pile and effective lever arm from the bottom of the pile.
d) Keulegan-Carpenter number (KC) has been introduced to find the value of drag coefficients and inertia coefficients.
e) Points to consider while computing wave force using Morison equation have been covered in detail.
f) Information regarding wave condition for design of breakwater, open sea jetty and wind forces have been included.
g) Annex A on dimensions of ships has been modified and revised with updated details.
h) Determination of wave forces and moments by Goda’s method has been included.

The composition of the Committee responsible for formulation of the standard is given in Annex E.

For the purpose of deciding whether a particular requirement of this standard is complied with the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 1960 ‘Rules for rounding off numerical values (*revised*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

PLANNING AND DESIGN OF PORTS AND HARBOURS — CODE OF PRACTICE

PART 3 LOADING

(Second Revision)

1 SCOPE

This standard (Part 3) deals with the loading on waterfront structures. It covers vertical imposed loads, horizontal forces due to berthing, bollard pulls, wave forces, currents and winds, reference is given to earthquake forces.

2 REFERENCES

The following standards contain provision which through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subjected to revision, and parties to agreement based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
875 (Part 3) : 2015	Code of practice for design loads (other than earthquake) for buildings and structures: Part 3 Wind loads (*third revision*)
1893 (Part 1) : 2016	Criteria for earthquake resistant design of structures: Part 1 General provision of buildings (*sixth revision*)
7314 : 1974	Glossary of terms relating to port and harbour engineering

3 TERMINOLOGY

For the purpose of this standard, the terms relating to tonnage of ships as listed below and those given in IS 7314 shall apply.

3.1 Gross Registered Tonnage (GRT) — It is broadly the capacity in cubic feet of the spaces within the hull, and of the enclosed spaces above the deck available for cargo, stores, passengers and crew, with certain exceptions, divided by 100. Thus 100 cubic feet of capacity is equivalent to 1 gross tonne.

3.2 Net Registered Tonnage (NRT) — It is derived from the gross tonnage by deducting spaces used for the accommodation of the master, officers, crew, navigation, propelling machinery and fuel.

3.3 Dead Weight Tonnage (DWT) — It is the weight in long tons (of 2 240 lb) of 1 016 kg cargo, stores, fuel, passengers and crew carried by the ship when loaded to her maximum summer load line.

3.4 Displacement Tonnage (DT) — The actual weight of the vessel, or the weight of water she displaces when afloat and may be either ‘loaded’ or ‘light’.

3.4.1 *Displacement ‘Loaded’* **—** The weight, in long tons (2 240 lb), of the ship and its contents when fully loaded with cargo, to the plimsoll mark or load line.

3.4.2 *Displacement ‘Light’* **—** The weight, in long tons, of the ship without cargo, fuel and stores.

4 SHIP CHARACTERISTICS

4.1 Relationship between the various tonnages are generally as given in Table 1.

4.1.1 For bulk carriers, relationship between *GRT* and *DWT* is generally as follows:

$$DWT = 1.649\ GRT + 1\ 462$$

4.1.2 For tankers, relationships between *DWT* and *DT* are generally as given in Table 2.

4.2 Ship Dimensions

For preliminary design purposes the ship dimensions given in Annex A may be used. For detailed design, ship dimensions appropriate to the type of service required may be obtained from a register of shipping. Llyods register of shipping has divided the design vessels into eight types and PIANC has prepared tables for the eight types of vessels with ship dimensions corresponding to 50 percent, 75 percent and 90 percent confidence limits.

5 DEAD LOADS

All dead loads of and on structures relating to docks and harbours should be assessed and included in the design.

Table 1 Ship Characteristics

(*Clause* 4.1)

Sl No.	Type of Ship	Gross Registered Tonnage (GRT)	Net Registered Tonnage (NRT)	Dead Weight Tonnage (DWT)	Displacement Tonnage
(1)	(2)	(3)	(4)	(5)	(6)
i)	Large sea going vessels	1	0.6	---	---
ii)	Small sea going vessels	1	0.4	---	---
iii)	Freighters	1	---	1.5	2
iv)	Large tankers	1	---	2	*See* **4.1.2**
v)	Large combined carriers	1	---	1.8	1.9
vi)	Large passenger ships	1	---	---	1
vii)	Passenger ships	1	---	1	---
viii)	Inland water way craft	1	0.8	---	---
ix)	Other types of ships	1	---	1.2	---

Table 2 Relationships between DWT and DT

(*Clause* 4.1.2)

Sl No.	DWT	DT/DWT Ratio
(1)	(2)	(3)
i)	25 000	1.32
ii)	50 000	1.26
iii)	80 000	1.25
iv)	100 000	1.20
v)	125 000	1.17
vi)	225 000 and above	1.15

6 IMPOSED LOADS

6.1 Vertical Imposed Loads

6.1.1 Surcharges due to stored and stacked material, such as general cargo, bulk cargo, containers and loads from vehicular traffic of all kinds, including trucks, trailers, railway, cranes, containers handling equipment and construction plant constitute vertical imposed loads.

6.1.2 *Truck Loading and Uniform Loading*

The berths shall be generally designed for the truck loading and uniform loading as given in Table 3.

Table 3 Truck Loading and Uniform Loading

(*Clause* 6.1.2)

Sl No.	Function of Berth	Truck Loading (IRC Class)	Uniform Vertical Imposed Loading T/m^2
(1)	(2)	(3)	(4)
i)	Passenger berth	B	1.0
ii)	Bulk unloading and loading berth	A	1 to 1.5
iii)	Container berth	A or AA or 70R	3 to 5
iv)	Cargo berth	A or AA or 70R	2.5 to 3.5
v)	Heavy cargo berth	A or AA or 70R	5 or more
vi)	Small boat berth	B	0.5
vii)	Fishing berth	B	1.0

NOTE — The relevant Indian Road Congress (IRC) publications may be referred for axle loads. The spacing of the loads may be changed to suit individual design requirements.

6.1.3 *Crane Loads*

Concentrated loads from crane wheels should be considered. An impact of 25 percent shall be added to wheel loads in the normal design of deck and stringers, 15 percent where two or more cranes act together, and

15 percent in the design of pile caps and secondary framing members. For other specialized mechanical handling equipment like reach stacker, fork lift and mobile harbour crane, etc impact effect shall be considered as per manufacturer's specification.

For geotechnical and structural design of piles, sheet piles and diaphragm walls, no impact shall be considered.

6.1.4 *Railway Loads*

Concentrated wheel loads due to locomotive wheels and wagon wheels shall be considered in accordance with the specification of the Indian Railways for the type of gauge and service at the locality in consideration.

6.1.5 For impact due to trucks and railways, one-third of the impact factors specified in the relevant standards may be adopted.

6.1.6 *Special Loads*

Special loads like pipeline loads or conveyor loads or exceptional loads, such as surcharge due to ore stacks, transfer towers, heavy machinery or any other type of heavy lifts should be individually considered.

6.1.7 When the imposed loads act on the fill behind the structure, such as in a sheet pile wharf so that the loads are transmitted to the structure through increased earth pressure, the retaining structure may be designed for uniformly distributed equivalent surcharge of half the value given in column 3 of Table 3. In cases where higher load intensity is expected, the actual value of surcharge may be taken.

6.1.8 If truck cranes are to be used in cargo handling, or if the backfill in a retaining structure is proposed to be placed with earth moving equipment of the crawler type, the uppermost portion of the waterfront structures, including the upper anchorage system should be designed according to the following loadings, whichever of the two is more unfavourable:

a) Imposed load of 6.0 tonne per square metre from back edge of the coping inboard for 1.5 m width.
b) Imposed load of 4.0 tonne per square metre from the back edge of the coping inboard for 3.5 m width.

6.1.9 *Container Crane*

The container terminal shall be designed to accommodate the largest vessel that can possibly arrive at the berth, considering the future growth of the port and access facilities, unless specifically restricted by the owner of the facility. The container crane size and wheel spacing shall also be determined based on the size of vessels arrived at. The wheel loads shall be based on the data of the manufacturer of such cranes. Normally, a set of manufacturers' data would have been available by the time the structural design of the wharf begins, and the information shall be based on this data. Crane operating condition shall consider load due to wind of 20 m/s in one of the four grid directions. When there are multiple cranes operating in a wharf, they should be considered to operate with a minimum spacing of 5 m between extreme wheels. Separate loading combinations shall be considered for parking the crane during extreme wind conditions.

The crane when loaded from the barge shall be on properly designed skid area on the jetty. The centre line of seaside crane line shall be more than 3 m from the face of the berth to avoid damages to the crane due to rolling of vessels.

6.1.10 *Mobile Harbour Crane*

If it is decided to have mobile cranes on the specific wharf, the numbers, type and capacity shall be finalied in the plannning stage. The load data shall be considered from manufacturers' information. The design shall consider adequate load conditions to ensure safety of the structure as per this code of practice for all scenarios of loading.

The pad loading from mobile harbour cranes shall be considered for the design of berth, especially for punching shear and bending, since the intensity due to the pad loading can be about 100 to 200 kN/m^2.

For detailed engineering load data and wheel mobile harbour crane, specification of crane manufacturers shall be sought.

6.2 Berthing Load

6.2.1 *Berthing Energy*

When an approaching vessel strikes a berth, a horizontal force acts on the berth. The magnitude of this force depends on the kinetic energy that can be absorbed by the fendering system. The reaction force for which the berth is to be designed can be obtained and deflection-reaction diagrams of the fendering system chosen. These diagrams are obtainable from fender manufacturers. The kinetic energy, E, imparted to a fendering system, by a vessel moving with velocity V is given by:

$$E = \frac{W_D \times V^2}{2g} \times C_m \times C_e \times C_s$$

where

W_D = displacement tonnage (DT) of the vessel, in tonne;
V = velocity of vessel, in m/s, normal to the berth (*see* **6.2.1.1**);
g = acceleration due to gravity, in m/s^2;
C_m = mass coefficient (*see* **6.2.1.2**);
C_e = eccentricity coefficient (*see* **6.2.1.3**); and

C_s = softness coefficient (*see* **6.2.1.4**).

NOTE — Ultimate energy shall be 1.4 times the energy E.

6.2.1.1 *Approach velocities*

Normal components of approach velocities of berthing vessels are recommended to be taken as given in Table 4. Berthing conditions will depend on alignment of the berth relative to currents, availability of tugs, physical layout of the harbour, wind and waves at time of berthing.

6.2.1.2 *Mass coefficient*

When a vessel approaches a berth and as its motion is suddenly checked, the force of impact which the vessel imparts comprises of the weight of the vessel and an effect from the water moving along with the vessel, such an effect, expressed in terms of weight of water moving with the vessel, is called the additional weight (W_A) of the vessel or the hydrodynamic weight of the vessel. The surge, sway, heave, roll, pitch and yaw motion of the body in the fluid will generate resistance of fluid against these motions and introduces a pressure field on the submerged surface. The component of force due to this pressure distribution in phase with body acceleration is the added mass, and the component of force in phase with body acceleration is the damping.

Table 4 Berthing Velocity of Ship with Tug Assistance

(*Clause* 6.2.1.1)

Sl No.	Velocity m/s	Ship Displacement (T)		
		Under 10 000	10 000 - 50 000	Over 50 000
(1)	(2)	(3)	(4)	(5)
i)	In favourable conditions	0.2	0.12	0.08
ii)	In moderate conditions	0.45	0.3	0.15
iii)	In unfavourable conditions	0.6	0.45	0.2

The berthing velocity of ship without tug boat assistance for vessels less than 20 000 tonne shall be as per the environmental and manoeuvring conditions and ship size.

a) The mass coefficient shall be calculated as follows:

$$C_m = 1 + \frac{2D}{B}$$

where

D = draught of the vessel, in m;

B = beam of the vessel, in m.

b) Alternative to (a) in case of a vessel which has a length much greater than its beam or draught generally for vessels with displacement tonnage greater than 20 000, the additional weight may be approximated to the weight of a cylindrical column of water of height equal to the length of vessel and diameter equal to the draught of vessel, then:

$$C_m = 1 + \frac{\pi/4\, D^2 L\omega}{W_D}$$

where

D = draught of the vessel, in m;

L = length of the vessel, in m;

ω = unit weight of water (1.03 tonne/m^3 – for sea water); and

W_D = displacement tonnage of the vessel, in tonnes.

6.2.1.3 *Eccentricity coefficient*

A vessel generally approaches a berth at an angle, denoted by and touches it at a point either near the bow or stern of the vessel. In such eccentric cases the vessel is imparted a rotational force at the moment of contact, and the kinetic energy of the vessel is partially expended in its rotational motion. Table 5 gives eccentricity coefficient values of l/r.

a) The eccentricity coefficient (C_e) is expressed as follows:

$$C_e = \frac{1 + (l/r)\sin^2\theta}{1 + (l/r)^2}$$

where

l = distance from the centre of gravity of the vessel to the point of contact projected along the water line of the berth, in m; and

r = radius of gyration of rotational radius on the plane of the vessel from its centre of gravity, in m.

b) The approach angle θ unless otherwise known with accuracy should be taken as 10°. For smaller vessels approaching wharf structures, the approach angle should be taken as 20°. For the vessels more than 20 000 DT, the angle shall be 6 degree.

c) The rotational radius of a vessel may be approximated L/4 to and, in normal case, the point of contact of the berthing vessel with the structure is at a point about L/4 from the bow or stern of the vessel, which is known as a quarter point contact. Also, if the approach angle θ is nearly 0°, then for large tankers, r = 0.2L and C_e = 0.4.

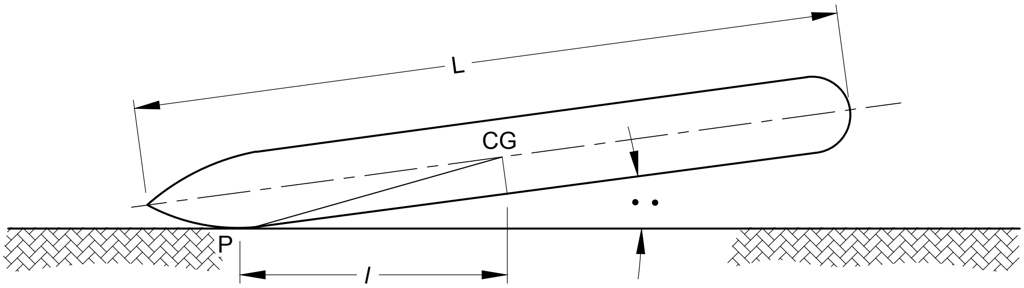


FIG. 1 VESSEL APPROACHING BERTH AT AN ANGLE.

Table 5 Values of Eccentricity Coefficient

[*Clause* 6.2.1.3 (*a*)]

l/r	Angle θ		
	0°	10°	20°
1	0.50	0.51	0.56
1.25	0.39	0.41	0.46

6.2.1.4 *Softness coefficient*

This softness coefficient (C_s) indicates the relation between the rigidity of the vessel and that of the fender, and hence also that between the energy absorbed by the vessel and by the fender. Since, the ship is relatively rigid compared with the usually yielding fendering systems, a value of 0.9 is generally applied for this factor, or 0.95, if higher safety margin is thought desirable.

6.2.2 High energy absorption is required during the mooring of very large vessels. However, the reaction force against the side of vessels should not exceed 400 kN/m^2 for first and second generation container ships, 250 to 300 kN/m^2 for oil tankers, 150 to 200 kN/m^2 for bulk carriers, gas tankers and very large crude carrier (VLCC); and less than 150 kN/m^2 for navy, coast guard and similar type of patrol vessels. Fender boards of appropriate size are provided to distribute the fendering force overa a larger area to satisfy this condition

6.2.2.1 Deflection-reaction diagram should give the berthing energy which the fender system can absorb. A fender system includes fenders and the berthing structure. The reaction force for the fenders and the structures will be the same.

6.2.3 Berthing load and, therefore, the energy of impact is to be considered for pier, dolphin and the like, with no backfill. In the case of continuous structures with backfill, this may not form a governing criterion for design, because of the enormous passive pressure likely to be mobilized. However, short lengths of gravity type, sheet pile type or relieving platform type berths may have to be checked for impact of vessels.

6.3 Mooring Loads

6.3.1 The mooring loads are the lateral loads caused by the mooring lines when they pull the ship into or along the dock or hold it against the forces of wind or current.

6.3.2 The wind speeds provided in IS 875 (Part 3) is the maximum wind speed observed in storm conditions and not 'crane operational wind speeds' (cranes stop working at around 20 m/s wind). The vessels are usually not moored for wind speed >30 m/s as the vessel movements during high wind may cause damage to the wharfs and vessel both. Mooring loads are to be calculated only for wind speed of about 30 m/s for mooring analysis and not for the storm wind as detailed in IS 875 (Part 3). The maximum mooring loads are due to the wind forces on exposed area on the broad side of the ship in light condition:

$$F = C_w A_w P$$

where

F = force due to wind, in kg;

C_w = shape factor = 1.3 to 1.6;

A_w = windage area, in m^2 (*see* **6.3.2.1**); and

P = wind pressure, in kg/m^2 to be taken in accordance with IS 875 (Part 3). The operational wind velocity shall be taken as per Fig. 1 of IS 875 (Part 3).

6.3.2.1 The windage area (A_w) can be estimated as follows:

$$A_w = 1.175\, L_p\, (D_M - D_L)$$

where

L_p = length between perpendicular, in m;

D_M = mould depth, in m; and

D_L = average light draft, in m.

NOTE — For windage area of a loaded container vessel as in this case the loaded scenario (with containers stacked on the deck of vessel) provides more windage area than unloaded vessel.

6.3.3 When the ships are berthed on both sides of a pier, the total wind force acting on the pier, should be increased by 50 percent to allow for wind against the second ship.

6.3.4 The appropriate load on the bollard shall then be calculated, which depends upon the layout of harbour, and position of bow line, stern line, spring line and breasting lines for guidance the bollard pulls independent of the number of laid-on hawsers, may be taken as given in Table 6 since the hawsers are not fully stressed simultaneously.

Table 6 Bollard Pulls

(*Clauses* 6.3.4 and 7.1)

Sl No.	Displacement Tonne	Line Pull Tonne
(1)	(2)	(3)
i)	2 000	10
ii)	10 000	30
iii)	20 000	60
iv)	50 000	80
v)	100 000	100
vi)	200 000	150
vii)	250 000	Greater than 200

NOTES

1 For ships of displacement tonnage 50 000 and over, the value of line pulls given above should be increased by 25 percent at quays and berths where there is a strong current.

2 Main bollards at the ends of individual large vessel berths at river structures should be designed for a line pull of 250 tonne for ships up to 100 000 tonne displacement and for double the values given above for larger ships.

6.3.5 The line pull will be towards the water and may make any angle to the longitudinal direction of the structure and is usually assumed to act horizontally.

6.3.6 In the design calculations of the bollard itself and its connections to the structure, line pull up to 30° and above the horizontal should be considered.

6.3.7 Pressure on the vessel as well as the structure due to the current should be taken into account, especially with a strong current and where the berth alignment deviates from the direction of the current. Determination of these forces is dealt with in **6.6**.

6.4 Differential Water Pressure

6.4.1 In the case of waterfront structures with backfill, the pressure caused by difference in water levels at the fill side and the waterside has to be considered in design. The magnitude of this hydrostatic pressure is influenced by the tidal range, free water fluctuations, the ground water influx, the permeability of the foundation soil and the structure as well as the efficiency of available backfill drainage.

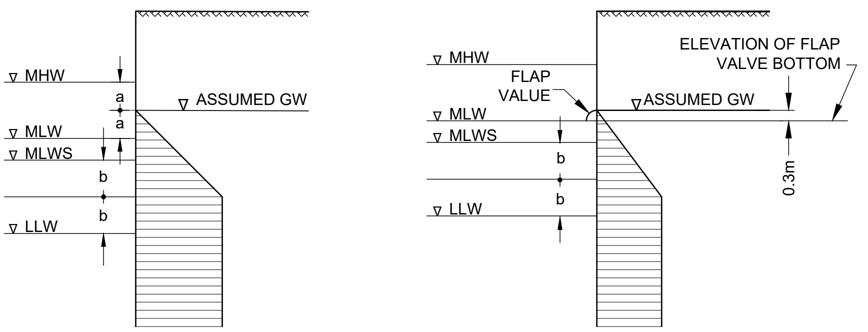
6.4.2 In the case of good and poor drainage conditions of the backfill the differential water pressure may be calculated on the guidelines given in Fig. 2. The level between MLWS and LLW is 'assumed LLW'.

6.5 Earthquake Forces

In areas susceptible to seismic disturbance, horizontal force equal to a fraction of the acceleration of gravity times the weight applied at its centre of gravity should be taken. The fraction will depend upon the likely seismic intensity of the area, and shall be taken in accordance with IS 1893 (Part 1). The weight to be used is the total dead load plus one-half of the imposed load.

6.6 Forces Due to Current

Pressure due to current will be applied to the area of the vessel below the water line when fully loaded. It is approximately equal to $v^2/2g$ per square metre of area,



2A POOR DRAINAGE CONDITION

2B GOOD DRAINAGE CONDITION

MHW	—	Mean high water
MEW	—	Mean low water
MLWS	—	Mean low water springs
LLW	—	Lowest low water
CW	—	Ground water

FIG. 2 GUIDE FOR CALCULATING DIFFERENTIAL WATER PRESSURE

where v is the velocity, in m/s and w is the unit weight of water, in tonne/m^3. A ship is generally berthed parallel to the current. With strong currents and where berth alignment materially deviates from the direction of the current, the likely force should be calculated by any recognized method and taken into account.

6.7 Wave Forces

6.7.1 As far as analysis and computation of forces exerted by waves on structures are concerned, there are three distinct types of waves, namely:

a) Non-breaking waves,

b) Breaking waves, and

c) Broken waves.

It is well recognized that similar wave condition gives rise to different wave forces depending on the form of wave breaking and distance from the structure. The theory used depends on whether structure is subjected to non-breaking (pulsating), impulse breaking (impact) or broken wave condition. The method of estimation of wave forces on structure depend on wave theories and type of the structure.

Appropriate wave theories (Linear/Airy's, Stokes 2nd order, Stokes 3rd order, Stokes 4th or 5th order, etc) shall be considered depending upon wave height, wave period and water depth. The breaking of waves shall be considered based on wave steepness (H/d or H/L, where H is wave height, d is water depth and L is wave length). The waves with wave steepness, $H/d \geq 0.78$ shall be considered as breaking waves in shallow water depths and $H/L \geq 0.14$ for deeper water depths (*see* Note). The significant wave height shall be taken from the following relationship:

$$\frac{H_m}{H_s} = 1.7 \text{ to } 1.9$$

where

H_m = maximum wave height; and

H_s = significant wave height.

NOTE — Special Publications like API-RP 2A-WSD-2005, 'Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms-Working Stress Design' may also be referred.

6.7.2 *Non-breaking Waves*

6.7.2.1 Generally, when the depth of water against the structure is greater than about 1.5 times the maximum expected wave height, non-breaking wave conditions occur.

6.7.2.2 Forces due to non-breaking waves are essentially hydrostatic. 'Sainflou Method' may be used for the determination of pressure due to non-breaking waves for simple wall in deep water. The method of computation using Sainflou Method is outlined in **B-1**. Goda's method is widely used for shallow or intermediate water for non-breaking and breaking waves (average force) on monolithic and composite structures and it is detailed in **B-2**.

The non-breaking wave forces on structures shall be estimated based on wave height, structure dimension and wave length. Morison equation shall be used for structures having $D/L < 0.2$ (where D is the diameter of the structure and L is the wave length). Froude Krylov forces or diffraction forces shall be estimated for structures with $D/L \geq 0.2$.

6.7.3 *Breaking Waves*

6.7.3.1 *Breaking waves cause both static and dynamic pressures*

6.7.3.2 Determination of the design wave for breaking wave conditions may be based on depth of water about seven breaker heights , seaward of the structure, instead of the water depth at which the structure is located.

6.7.3.3 The actual pressures caused by a breaking wave is obtained by following the method suggested by Minikin, whose method of computation is outlined in Annex C.

6.7.4 *Broken Waves*

6.7.4.1 Locations of certain structures like protective structure will be such that waves will break before striking them. In such cases, no exact formulae have been developed so far to evaluate the forces due to broken waves, but only approximate methods based on certain simplifying assumptions are available and these are given in Annex D.

6.7.5 *Wave Forces on Vertical Cylindrical Structures, such as Piles*

6.7.5.1 The total force *'F'* exerted by non-breaking waves on cylindrical piles can be divided into two components:

a) Force due to inertia, and

b) Force due to drag.

The total force acting on the vertical pile is the summation of inertial force and drag force.

$$F_T = \int_{-d}^{0} (f_I + f_D)dz$$

$$F_I = \int_{-d}^{0} f_I dz = \frac{w\rho g C_D H^2 D}{8} \tanh(kd)\sin(\theta)$$

$$F_D = \int_{-d}^{0} f_D dz = \rho g C_D H^2 D \left[\frac{2kd + \sinh(2kd)}{16\sinh(2kd)}\right] \cos(\theta) \left|\cos(\theta)\right|$$

The maximum total force acting on the vertical pile is given by:

$$F_M = F_{IM} \sin(\theta_{max}) + F_{DM} \cos(\theta_{max}) \left|\cos(\theta_{max})\right|$$

The maximum force occurs at a phase,

$$\sin(\theta_{max}) = \frac{F_{1M}}{2F_{DM}}$$

$$\frac{F_{1M}}{\rho g C_D H^2 D} = \frac{w}{8}\tanh(kd)$$

$$\frac{F_{DM}}{\rho g C_D H^2 D} = \left[\frac{2kd + \sinh(2kd)}{16\sinh(2kd)}\right]$$

$$M_M = F_{1M}\sin(\theta_{max})S_l + F_{DM}\cos(\theta_{max})\left|\cos(\theta_{max})\right| S_D$$

$$S_l = d\left[1 - \frac{\cosh(kd) - 1}{kd\sinh(kd)}\right]$$

$$S_D = d\left[1 - \frac{1}{2n}\left(\frac{\cosh(2kd) - 1 + 2(kd)^2}{2\sinh(2kd)kd}\right)\right]$$

If $\frac{F_{IM}}{2F_{DM}} > 1$, the force is predominantly inertial. The maximum force is taken as 1.25 F_{IM}.

The effective lever arm from the bottom of the pile is,

$$S = \frac{M_M}{F_M}$$

The drag coefficients and inertia coefficients depends upon the value of Keulegan–Carpenter number (KC),

$$KC = \frac{\pi H}{D}\coth(kd)$$

where

F_T = total force on vertical pile from the sea bottom to the surface crest elevation, in N;

f_I = inertia force on vertical pile at any section of the pile, in N/m;

f_D = drag force on vertical pile at any section of the pile, in N/m;

F_I = total inertial force on vertical pile from the sea bottom to the surface crest elevation, in N;

F_D = total drag force on vertical pile from the sea bottom to the surface crest elevation, in N;

θ = wave phase, in radians.

k = wave number $= \frac{2\pi}{L}$, in m^{-1}

L = wavelength $= \frac{gT^2}{2\pi}\tanh(kd)$, in m; the wavelength is obtained by iterative process.

$$w = \frac{C_D D}{C_M H}$$

C_D = drag force coefficient;

C_M = inertial force coefficient;

D = diameter of pile, in m;

H = wave height, in m;

T = time period of the wave, in s;

d = water depth, in m;

z = water depth at any level from still water line, in m;

ρ = density of sea water = 1025 kg/m^3;

g = acceleration due to gravity = 9.81 kg/m^2;

F_M = maximum total force on vertical pile from the sea bottom to the surface crest elevation, in N;

F_{IM} = total maximum inertia force on vertical pile from the sea bottom to the surface crest elevation, in N;

F_{DM} = total maximum drag force on vertical pile from the sea bottom to the surface crest elevation, in N;

θ_{max} = wave phase at which the total force is maximum, in radians;

M_M = maximum total moment, in Nm;

S_D = effective lever arm for F_{DM} from the bottom of pile, in m;

n = ratio of group celerity to celerity of wave

$$= \frac{1}{2}\left(1 + \frac{2kd}{\sinh(2kd)}\right)$$

S_I = effective lever arm for F_{IM} from the bottom of pile, in m; and

S = effective lever arm for F_M from the bottom of pile, in m.

The above equation is applicable only when $\frac{D}{L} < 0.2$ and $\frac{H}{D} > 1$.

The wave force can also be computed using Morison equation considering the following:

a) The operating (1-year return period) and survival sea state (100 year or 50 year return period depending on life of structure to estimate the wave force and 1 000 year return period to fix the top level) shall be considered. The wave slam and slap on structures shall also be considered if the air gap provided is less than 1.5 m.

b) The wave force reduction factor to account for three dimensional effects of wave spreading in reality versus 2D wave theory can be considered along with,

1) wave load reduction due to array of piles spaced at less than 3 times diameter of piles (shielding effect).
2) wave load on rectangular structures especially partly submerged, such as breasting and mooring dolphins.

c) In lieu of C_d and C_m variation (*see* Fig. 3) along the depth as a function of KC, wave load on piles (assuming KC values may range from 20 to 30 for most practical conditions across the depth), it is recommended to use the following coefficients in Morison equation:

C_d = 0.7 and C_m = 2.0, for piles without marine growth

C_d = 1.05 and C_m = 2.0, for piles with marine growth

The Morison equation can also be used for estimating force on piles due to breaking wave with the assumption that the wave acts on the pile as a water mass with high velocity with zero acceleration. Hence C_m is set to zero and C_d is increased to 1.75.

The wave and current shall be assumed to act collinear and the combined force shall be calculated using the vectoral sum velocity due to wave and current.

NOTE — Addition of wave and current force calculated separately and added is not correct due to non-linear term in the drag part of the Morison equation.

6.7.5.2 Tests have indicated that the wave forces are smallest for a cylindrical section, increasing for flat or irregular surfaces such as concrete and H-pipes.

6.7.6 *Wave Condition for Design of Breakwater*

The depth limited wave breaking shall be considered for shallow water to estimate significant wave height H_s, (0.5 times design water depth at seaside toe of the breakwater), whereas the significant wave height with a return period of 100 years to be considered for intermediate and deep water. A minimum height of 0.5 m shall be provided above wave run-up + strom surge + high tide level for fixing the crest elevation.

The significant wave height shall be multiplied by the following factor to obtain the design wave height for different type of structures:

Erosion protection	: 1.0
Rubble mound structures	: 1.0 to 1.3
Concrete breakwater	: 1.6 to 1.8
Berth structures	: 1.6 to 1.8

The operating wave condition during which the construction will be carried out shall be specified and suitable protection with under armour shall be part of the design.

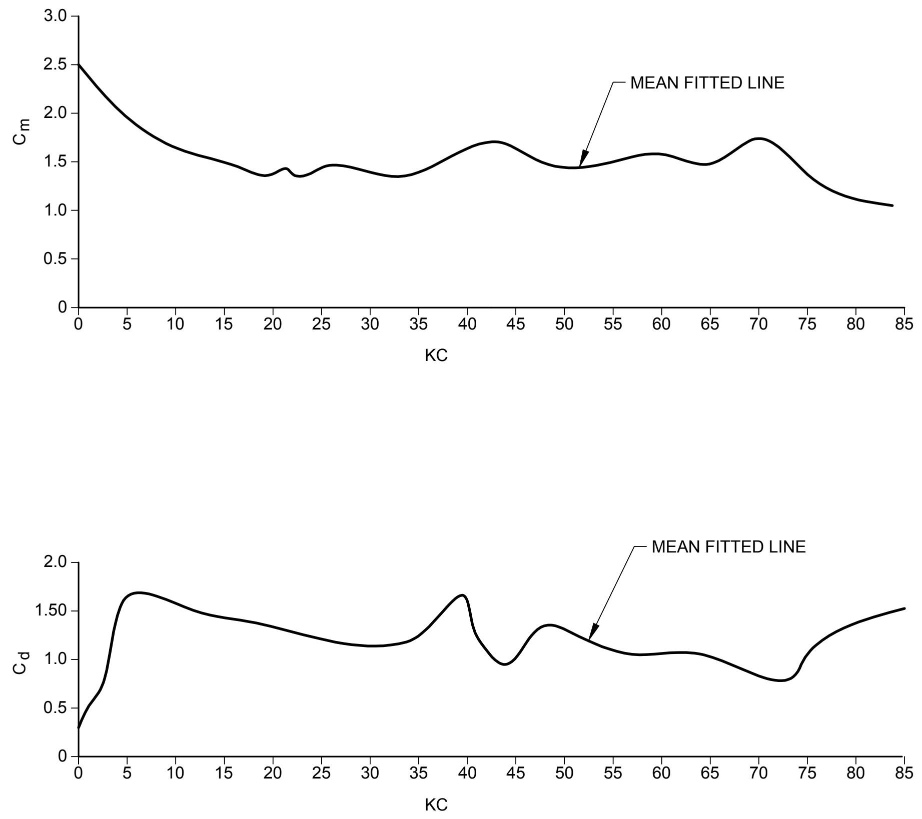


FIG. 3 VARIATION OF C_D AND C_M BASED ON KC

6.7.7 *Open Sea Jetty*

The wave force shall be calculated based on the maximum wave height with appropriate return period where as significant wave height shall be considered along with run up for large diameter piles to fix the top level. High tide level along with storm surge/Tsunami shall be accounted using independent or associated method with about 1.5 m air gap.

The recommended return period is 100 years for wave force estimation and 1 000 years for fixing the top level.

6.8 Wind Forces

Wind forces on structures shall be taken in accordance with IS 875 (Part 3) as applicable.

7 COMBINED LOADS

The combination of loadings for design is dead load, vertical imposed loads, plus either berthing load, or line pull, or earthquake or wave pressure. If the current and alignment of the berth are likely to give rise to line pull in excess of that given in Table 4, provisions for such extra pull in combination of likely wind should be made. The worst combination should be taken for design.

The mooring line force due to oscillations of vessels either due to wave and current in an indirect direction or by passing vessel nearby shall be considered. The design of bollards shall be safe till mooring line breaks and the structure shall be safe till bollard fails.

ANNEX A

(*Clause* 4.2)

DIMENSIONS OF SHIPS

A-1 GENERAL CARGO SHIP

The representative dimensions of the cargo ship are as under:

Dead Weight Tonnage (T)	*Displacement* (T)	*Length Overall* (m)	*Length Between Perpendicular* (m)	*Breadth* (m)	*Depth* (m)	*Maximum Draft* (m)
1 000	1 580	63	58	10.3	5.2	3.6
2 000	3 040	78	72	12.4	6.4	4.5
3 000	4 460	88	82	13.9	7.2	5.1
5 000	7 210	104	96	16.0	8.4	6.1
7 000	9 900	115	107	17.6	9.3	6.8
10 000	13 900	128	120	19.5	10.3	7.6
15 000	20 300	146	136	21.8	11.7	8.7
20 000	26 600	159	149	23.6	12.7	9.6
30 000	39 000	181	170	26.4	14.4	10.9
40 000	51 100	197	186	28.6	15.7	12.0

A-2 PASSENGER SHIP

The representative dimensions of the passenger ship are as under:

Dead Weight Tonnage (T)	*Displacement* (T)	*Length Overall* (m)	*Length Between Perpendicular* (m)	*Breadth* (m)	*Depth* (m)	*Maximum Draft* (m)
1 000	850	60	54	11.4	4.1	1.9
2 000	1 580	76	68	13.6	5.3	2.5
3 000	2 270	87	78	15.1	6.2	3.0
5 000	3 580	104	92	17.1	7.5	3.6
7 000	4 830	117	103	18.6	8.6	4.1
10 000	6 640	133	116	20.4	9.8	4.8
15 000	9 530	153	132	22.5	11.5	5.6
20 000	12 300	169	146	24.2	12.8	7.6
30 000	17 700	194	166	26.8	14.9	7.6
50 000	27 900	231	197	30.5	18.2	7.6
70 000	37 600	260	220	33.1	20.7	7.6

A-3 FERRY

The representative dimensions of ferries are as under:

Dead Weight Tonnage (T)	*Displacement* (T)	*Length Overall* (m)	*Length Between Perpendicular* (m)	*Breadth* (m)	*Depth* (m)	*Maximum Draft* (m)
1 000	810	59	54	12.7	4.6	2.7
2 000	1 600	76	69	15.1	5.8	3.3
3 000	2 390	88	80	16.7	6.5	3.7
5 000	3 940	106	97	19.0	7.6	4.3
7 000	5 480	119	110	20.6	8.5	4.8
10 000	7 770	135	125	22.6	9.5	5.3
15 000	11 600	157	145	25.0	10.7	6.0
20 000	15 300	174	162	26.8	11.7	6.5
30 000	22 800	201	188	29.7	13.3	7.4
40 000	30 300	223	209	31.9	14.5	8.0

A-4 ROLL-ON/ROLL-OFF SHIP

The representative dimensions of the roll-on-roll-off ship are as under:

Dead Weight Tonnage (T)	*Displacement* (T)	*Length Overall* (m)	*Length Between Perpendicular* (m)	*Breadth* (m)	*Depth* (m)	*Maximum Draft* (m)
1 000	1 970	66	60	13.2	5.2	3.2
2 000	3 730	85	78	15.6	7.0	4.1
3 000	5 430	99	90	17.2	8.4	4.8
5 000	8 710	119	109	19.5	10.5	5.8
7 000	11 900	135	123	21.2	12.1	6.6
10 000	16 500	153	141	23.1	14.2	7.5
15 000	24 000	178	163	25.6	16.9	8.7
20 000	31 300	198	182	27.4	19.2	9.7
30 000	45 600	229	211	30.3	23.0	11.3

A-5 BULK CARRIER

The representative dimensions of bulk carrier are as under:

Dead Weight Tonnage (T)	*Displacement* (T)	*Length Overall* (m)	*Length Between Perpendicular* (m)	*Breadth* (m)	*Depth* (m)	*Maximum Draft* (m)
5 000	6 740	106	98	15.0	8.4	6.1
7 000	9 270	116	108	16.6	9.3	6.7
10 000	13 000	129	120	18.5	10.4	7.5
15 000	19 100	145	135	21.0	11.7	8.4
20 000	25 000	157	148	23.0	12.8	9.2
30 000	36 700	176	167	26.1	14.4	10.3
Handymax 40 000 to 50 000	45 000 to 60 000	150 to 200	135 to 180	28 to 32	14 to 16	10 to 12
50 000	59 600	204	194	32.3	16.8	12.0
70 000	81 900	224	215	32.3	18.6	13.3
100 000	115 000	248	239	37.9	20.7	14.8
150 000	168 000	279	270	43.0	23.3	16.7
200 000	221 000	303	294	47.0	25.4	18.2
250 000	273 000	322	314	50.4	27.2	19.4

A-6 CONTAINER VESSELS

The dimensions of container vessels are as under:

Dead Weight Tonnage (T)	*Displacement* (T)	*Length Overall* (m)	*Length Between Perpendicular* (m)	*Breadth* (m)	*Depth* (m)	*Maximum Draft* (m)
7 000	10 200	116	108	19.6	9.3	6.9
10 000	14 300	134	125	21.6	10.7	7.7
15 000	21 100	157	147	24.1	12.6	8.7
20 000	27 800	176	165	26.1	14.1	9.5
25 000	34 300	192	180	27.7	15.4	10.2
30 000	40 800	206	194	29.1	16.5	10.7
40 000	53 700	231	218	32.3	18.5	11.7
50 000	66 500	252	238	32.3	20.2	12.5
Panamax 52 500	68 500	289	260	32.31	20	12.04
60 000	79 100	271	256	35.2	21.7	13.2
Suezmax 160 000	208 000	400	370	50	30	20.1

The most commonly used is the panamax plus vessel, for 54 000 DWT. The average size is 4 300 - 4 600 TEU (twenty foot equivalent unit), Length is about 270 – 300 m, draft is about 11-12 m and breadth is about 38 - 40 m. The typical size of a 12 000 TEU vessel is length 366 m, beam 51 m, draft 13 m and 152 000 displacement tons.

A-7 TANKER DIMENSIONS

The dimensions of tankers are as under:

Dead Weight Tonnage (T)	*Displacement* (T)	*Length Overall* (m)	*Length Between Perpendicular* (m)	*Breadth* (m)	*Depth* (m)	*Maximum Draft* (m)
1 000	1 450	59	54	9.7	4.3	3.8
2 000	2 810	73	68	12.1	5.4	4.7
3 000	4 140	83	77	13.7	6.3	5.6
5 000	6 740	97	91	16.0	7.5	6.1
7 000	9 300	108	102	17.8	8.4	6.7
10 000	13 100	121	114	19.9	9.5	7.5
15 000	19 200	138	130	22.5	11.0	8.4
20 000	25 300	151	143	24.6	12.2	9.1
30 000	37 300	171	163	27.9	14.0	10.3
50 000	60 800	201	192	32.3	16.8	11.9
70 000	83 900	224	214	36.3	18.9	13.2
100 000	118 000	250	240	40.6	21.4	14.6
150 000	174 000	284	273	46	24.7	16.4
200 000	229 000	311	300	50.3	27.3	17.9
300 000	337 000	354	342	57	31.5	20.1

A-8 MOORING EQUIPMENT FOR DIFFERENT SHIP SIZES

Typical mooring line are as given below:

Ship Size (DWT) (1)	*Mooring Equipment* (2)
25 000	14 nos. polypropylene dia 60 mm
75 000	20 nos. polypropylene dia 72 mm
14 000	20 nos. polypropylene dia 80 mm
250 000	24 nos. polypropylene dia 88 mm
550 000	20 nos. steel dia 42 mm + 2 nos. propylene dia 80 mm

ANNEX B

(*Clause* 6.7.2.2)

SAINFLOU METHOD

B-1 FORMATION OF CLAPOTIS

B-1.1 Suppose a wave of length L and height H strikes the vertical AC, a standing wave or clapotis is formed, features of which are given as:

$$h_0 = \frac{\pi H^2}{L} \coth \frac{2\pi d}{L}$$

$$P_1 = \frac{wH}{\cosh \frac{2\pi d}{L}}$$

The following symbols are indicated in Fig. 4.

d = depth from still water level

H = Height of original free wave

L = length of wave

w = weight per m^3 of water

P_1 = pressure, the clapotis adds to or substracts from still water pressure

h_o = height of orbit centre (on mean level) above still water level

NOTE — Plotted graphs are available giving values of Lh_o and P_1 corresponding to various values of d/L ratio, from which values of h_o and P_1 can be readily obtained.

B-1.2 Assuming the same still water level on both sides of the wall, the pressure diagram will be as given in Fig. 5, in which :

$$R_e = \frac{(d+H+h_o)(wd+P_1)}{2} - \frac{wd^2}{2}$$

$$M_e = \frac{(d+H+h_o)^2(wd+P_1)}{6} - \frac{wd^3}{6}$$

$$R_i = \frac{wd^2}{2} - \frac{(d+h_o-H)(wd-P_1)}{2}$$

$$R_i = \frac{wd^3}{6} - \frac{(d+h_o-H)(wd-P_1)}{6}$$

where

R_e = resultant pressure with maximum crest level;

M_e= moment due to R_e about the base;

R_i = resultant pressure with minimum through level; and

M_i = moment due to R_i about the base.

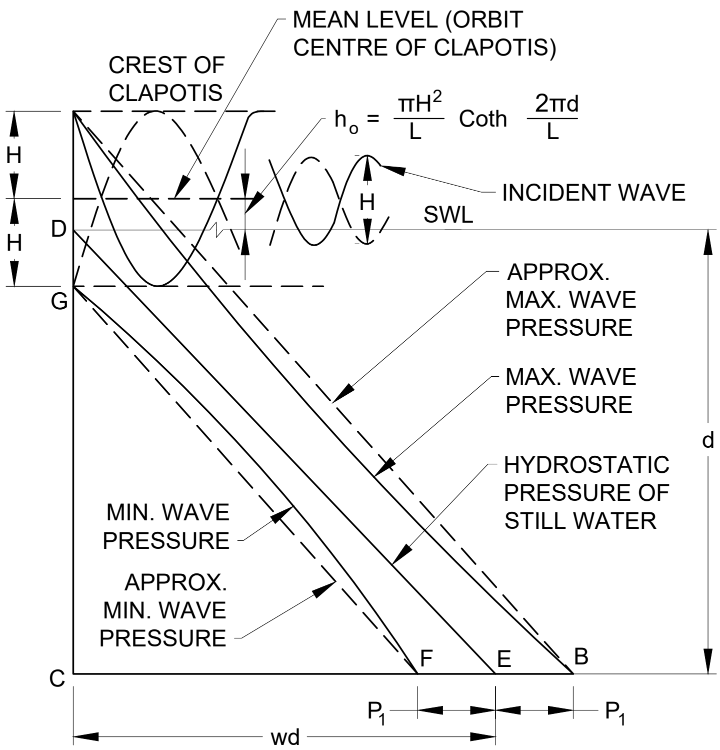


FIG. 4 CLAPOTIS ON VERTICAL WALL

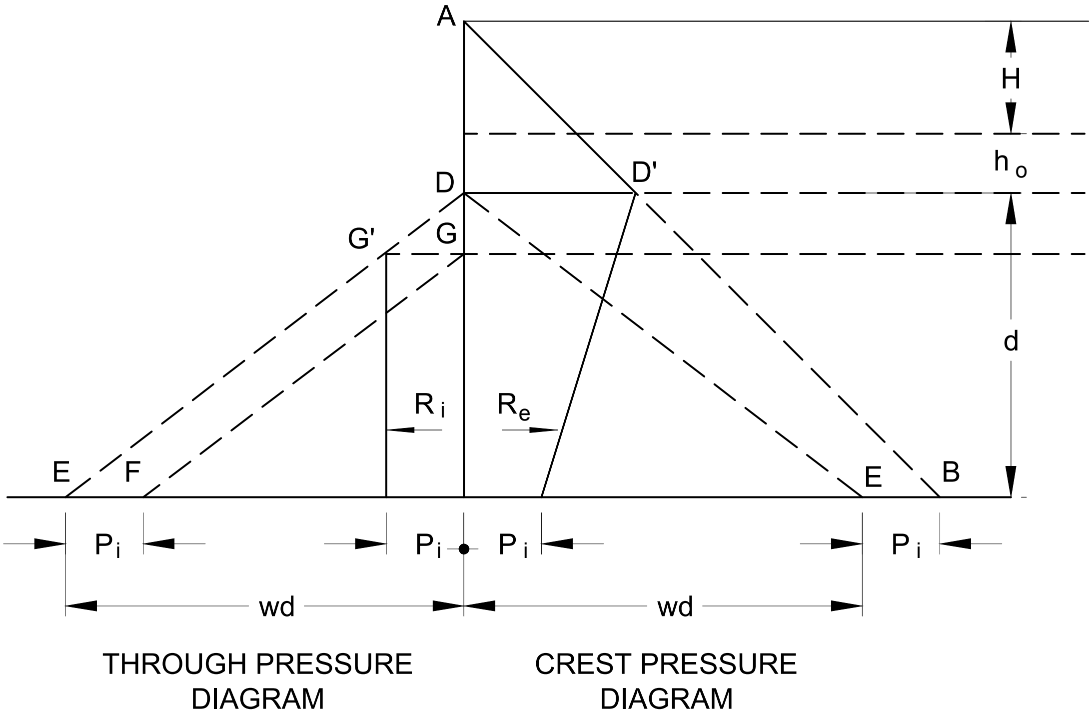


FIG. 5 SAINFLOU WAVE PRESSURE DIAGRAM

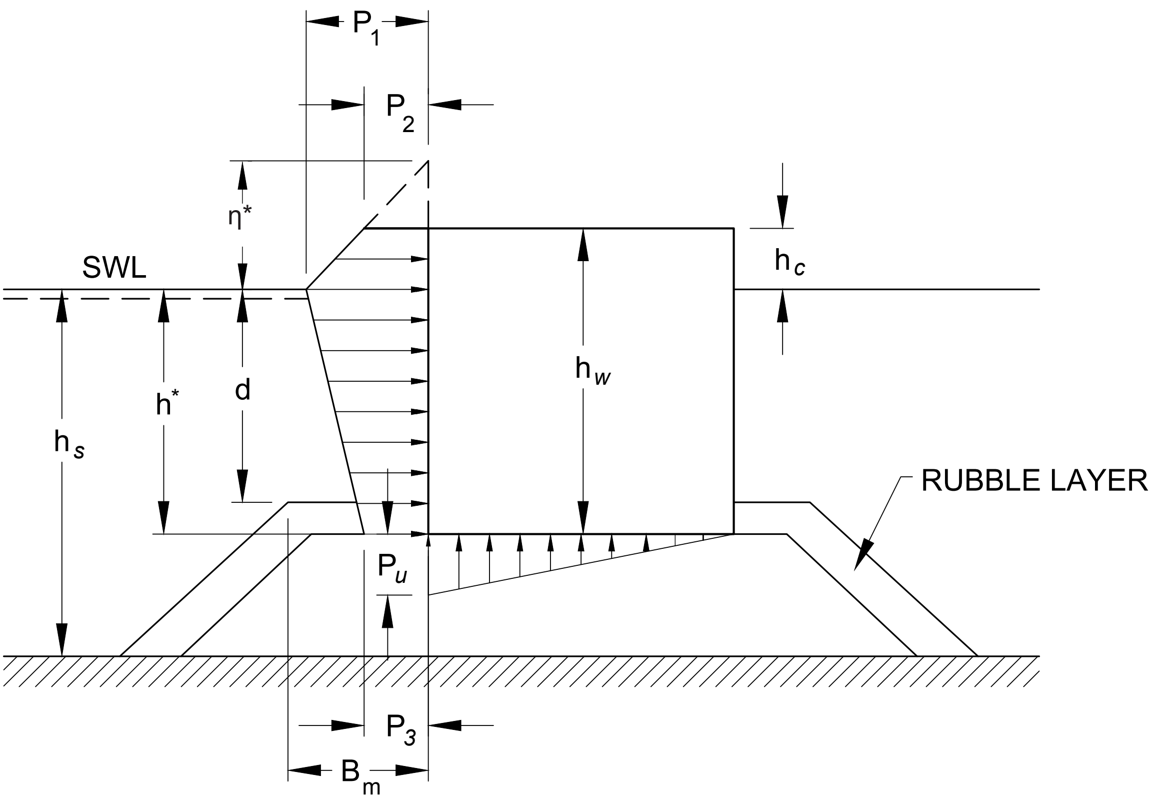


FIG 6. PRESSURE DIAGRAM

B-2 GODA'S METHOD

Goda's method may underestimate the wave force, that is, non-breaking wave forces are acting on the wall are assumed. A factor of safety of 1.2 is recommended for structure design against sliding and overturning. *See also* Fig. 6, in which :

$\eta^* = 0.75\ (1+\cos\beta)\ \lambda_1 H_{design}$

$P_1 = 0.5\ (1+\cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_1\cos^2\beta)\,\rho_w g\, H_{design}$

$$P_2 = \begin{cases} \left(1-\frac{hc}{\eta^*}\right)P_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* <= h_c \end{cases}$$

$P_3 = \alpha_3\, P_1$

$P_4 = 0.5\ (1+\cos\beta)\ \lambda_3\alpha_1\alpha_3\rho_w\, gH_{design}$

where

$\beta =$ angle of incidence of waves (angle between wave crest and front of structure); and

$H_{\text{design}} =$ design wave height defined as the highest wave in the design sea state at a location just in front of the breakwater. If seaward of a surf zone Goda (1985) recommends for practical design a value of 1.8 H_s to be used corresponding to the 0.15 percent exceedance value for Rayleigh distributed wave heights. This corresponding to $H_{1/250}$ (mean of the height of the waves included in 1/250 of the total number of waves, counted in descending order of height from the highest wave).

$$\alpha_1 = 0.5 + 0.5\left[\frac{4\pi\frac{h_c}{L}}{sinh 4\pi\frac{h_c}{L}}\right]^2$$

$$\alpha_2 = \text{the smallest of } \frac{h_b - d}{3h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}}$$

$$\alpha_3 = 1 - \frac{h_w - h_c}{h_b}\left[1 - \frac{1}{\cosh(2\pi hs/L)}\right]$$

where

$L =$ wave length at water depth hb corresponding to that of the significant wave; and

$h_b =$ water depth at a distance of $5h_s$ seaward of the breakwater front wall.

λ_1, λ_2 *and* λ_3 are modification factors depending on the structure type. For conventional vertical wall structure, $\lambda_1 = \lambda_2 = \lambda_3 = 1$.

Step by step procedure:

a) Select a design sea state and identify the significant wave height H_s and significant wave period T_s.

b) Determine h by $h = D + 5 \times m\ H_s$, where is the bottom slope.

c) Now calculate the non-breaking wave height, H_b at h. Note H_b should be greater than maximum wave height. The maximum wave height may be estimated as 1.8 times the H_s.

d) Now calculate wave length 'L' for depth 'D' using significant wave period of the design sea state.

e) Calculate the wave forces and moments using the Goda's equation given above.

B-2.1 When there is no water on the landward side of the wall, then the total pressure on the wall will be represented by the triangle ACB (*see* Fig. 4) when the clapotis crest is at A.

B-2.2 If there is wave action on the landward side also, then the condition of crest of clapotis on the seaside and trough of the wave on the harbour side will produce maximum pressure from the seaside.

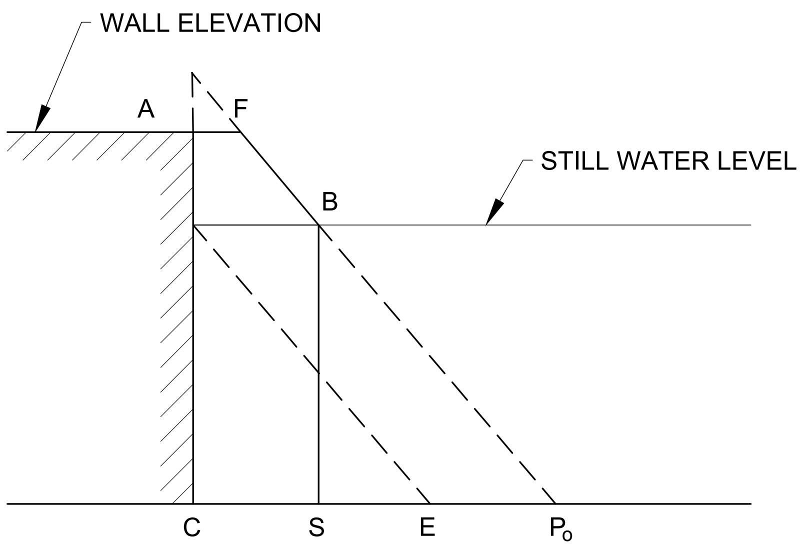


FIG. 7 PRESSURE ON WALLS OF LOW HEIGHT

The maximum pressure from the harbour side will be produced when the trough of the clapotis on the seaside and the crest of wave on the land-side are at the structure

B-2.3 Wall of Low Height

If the height of the wall is less than the predicted wave-height at the wall, forces may be approximated by drawing the force polygon as if the wall were higher than the impinging waves then analyzing only that portion below the wall crest.

Forces due to a wave crest at the wall are computed from the area *AFBSC,* as shown in Fig. 7.

ANNEX C

(*Clause* 6.7.3.3)

MINIKIN'S METHOD

C-1 FORCE DUE TO BREAKING WAVES

Pressure caused by breaking waves is due to a combination of dynamic and hydrostatic pressures as given below:

a) The dynamic pressure is concentrated at still water level and is given by:

$$P_{\mathrm{m}} = 101\frac{H_{\mathrm{b}}wd}{L_{\mathrm{D}}D}(D+d)$$

where

P_{m} = dynamic pressure, in kg/m^2;

H_{b} = height of wave just breaking on the structures, in m;

w = unit weight of the water, in kg/m^3;

d = depth of water at the structure, in m;

D = deeper water depth, in m; and

L_{D} = deeper water length, in m.

Values of L_{D} and D may be computed by accepted methods (*see* Note).

NOTE — Reference may be made to 'Shore Protection, Planning and Design', Technical Report No. 4 (third edition), US Army Coastal Engineering Research Centre.

b) The hydrostatic pressure P_{s} on the seaward side at still water level and the pressure P_{d} at the depth, d, are given by

$$P_{\mathrm{s}} = \frac{wH_{\mathrm{b}}}{2}$$

$$P_{\mathrm{d}} = w\left(d + \frac{H_{\mathrm{b}}}{2}\right)$$

For explanation of symbols, *see* Fig. 8.

C-2 CALCULATION OF FORCE AND MOMENT

C 2.1 The Minikin wave pressure diagram is given in Fig. 8.

C-2.1.1 *With Water on Land Side*

The resultant wave thrust R on structure per linear metre of structure is determined from the area of pressure diagram and is given by:

$$R = \frac{P_{\mathrm{m}}H_{\mathrm{b}}}{3} + P_s\left(d + \frac{H_{\mathrm{b}}}{4}\right)$$

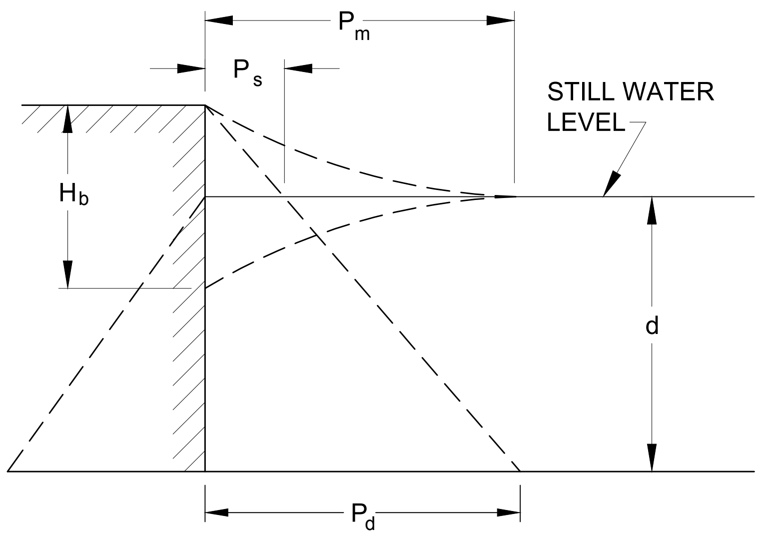


FIG. 8 MINIKIN WAVE PRESSURE DIAGRAM

The resultant overturning moment M about the ground line before the wall is the sum of the moments of the individual area and is given by:

$$M = \frac{P_m H_b}{3} + \frac{P_s d^2}{2} + \frac{P_s H_b}{4}\left(d + \frac{H_b}{6}\right)$$

For explanation of symbols, *see* Fig. 8.

C-2.1.2 *With No Water on Land Side*

Thrust R per linear metre is given by:

$$R = \frac{P_m H_b}{3} + \frac{P_d}{2}\left(d + \frac{H_b}{2}\right)$$

Moment M about the ground line is given by:

$$M = \frac{P_m H_b}{3} d + \frac{P_d}{6}\left(d + \frac{H_b}{2}\right)^2$$

For explanation of symbols, *see* Fig.8.

ANNEX D

(*Clause* 6.7.4.1)

BROKEN WAVES

D-1 WALL SEA WARD OF SHORELINE

Such walls are subjected to wave pressure which are partly dynamic and partly static (*see* Fig. 9)

Dynamic part of the pressure P_m will be,

$$P_m = \frac{wd_b}{2}$$

where

w = unit weight of water, in kg/m^3; and

d_b = breaking wave depth, in m.

The static part will vary from zero at a height h_c, where h_c is the height of that portion of the breaking wave above still water level which is given by:

$$h_c = 0.7\ h_b$$

to the maximum static pressure at the wall base and this maximum pressure P_s which will be given by:

$$P_s = w\ (d + h_c)$$

where

d = depth of water at structure, in m.

Assuming that the dynamic pressure is uniformly distributed from the still water level to a height, h_c, above the still water level, the total wave thrust R will be:

$$R = R_m + R_s$$

$$P_m h_c + P_s\left(\frac{d + h_c}{2}\right)$$

$$= \frac{wd_b\ h_c}{2} + \frac{w}{2}\left(d + h_c\right)^2$$

The overturning moment M, about the ground line at the seaward face of the structure will be:

$$M = M_m + M_s$$

$$= R_m\left(d + \frac{h_c}{2}\right) + R_s\left(\frac{d + h_c}{3}\right)$$

$$= \frac{wd_b h_c}{2}\left(d + \frac{h_c}{2}\right) + \frac{w}{6}\left(d + h_c\right)^3$$

where

w = unit weight of water, in kg/m^3

For explanation of other symbols, *see* Fig. 9.

D-2 WALL LANDWARD OF SHORELINE

Wave pressure diagram in this case will be given in Fig. 10.

$$\text{Dynamic pressure } P_m = \frac{wd_b}{2}\left[1 - \frac{X}{X_2}\right]^2$$

$$\text{Static pressure } p_s = wh' = wh_c\left[1 - \frac{X_1}{X_2}\right]$$

$$\text{Wave thrust} = R = R_m + R_s$$

$$= P_m h' + P_s h'$$

$$= \frac{wd_b h_c}{2}\left[1 - \frac{X_1}{X_2}\right]^2 + \frac{wh_c^2}{2}\left[1 - \frac{X_1}{X_2}\right]$$

$$\text{Moment } M = M_m + M_s$$

$$= R_m \frac{h'}{2} + R_s \frac{h'}{3}$$

$$= \frac{wd_b h_c^2}{4}\left[1 - \frac{X_1}{X_2}\right] + \frac{wh_c^3}{6}\left[1 - \frac{X_1}{X_2}\right]^3$$

where

w = unit weight of water,

For explanation of symbols, *see* Fig. 10.

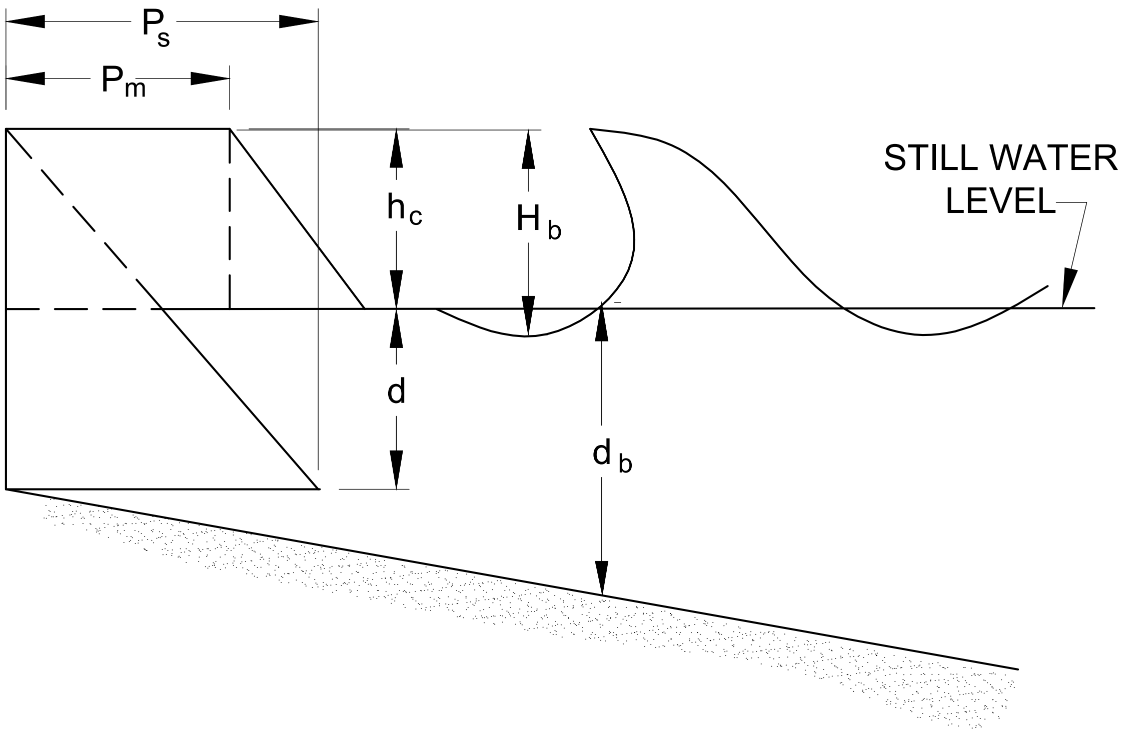


FIG. 9 WAVE PRESSURES FROM BROKENWAVES WALL SEAWARDOF SHORELINE

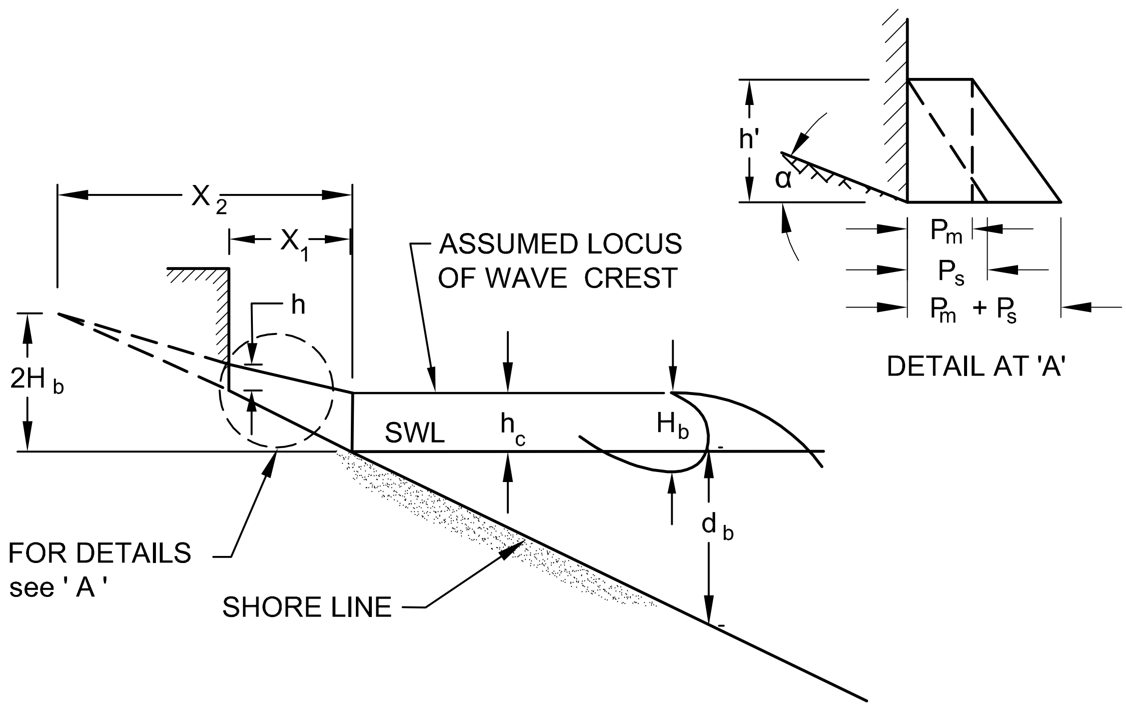


FIG. 10 WAVE PRESSURES FROM BROKEN WAVES: WALL LANDWARD OF SHORELINE

ANNEX E

(*Foreword*)

COMMITTEE COMPOSITION

Ports and Harbours Sectional Committee, CED 47

Organization	*Representative(s)*
In personal capacity (*3353, Sector D, Pocket 3, Vasant Kunj, New Delhi 110 070*)	Dr M. Hariharan (***Chairman***)
Afcons Infrastructure Limited, Mumbai	Shri V. Ramamurty Ms Vasumitha Joshi (*Alternate*)
Central Institute of Coastal Engg. For Fishery, Bengaluru	Shri N. Venkatesh Prasad Shri M. B. Belliappa (*Alternate*)
Central Water & Power Research Station, Pune	Dr Prabhat Chandra Dr J. D. Agrawal (*Alternate*)
Chennai Port Trust, Chennai	Superintending Engineer (Civil) Executive Engineer (Civil) (*Alternate*)
CSIR-National Institute of Oceanography, Panaji	Shri D. Ilangovan
CSIR-Structural Engineering Research Centre, Chennai	Dr J. Rajasankar Shri P. Gandhi (*Alternate*)
Engineers India Limited, New Delhi	Shri Bhaskar Pal Shri Charanjit Singh (*Alternate*)
Gujarat Maritime Board, Gandhinagar	Chief Engineer Deputy Chief Engineer (*Alternate*)
India Meteorological Department, New Delhi	Dr S. K. Peshin Dr S. D. Attri (*Alternate*)
Indian Institute of Technology Madras, Chennai	Prof R. Sundaravadivelu Prof S. A. Sannasiraj (*Alternate*)
ITD Cementation India Ltd, Kolkata	Shri Prodyot Kumar Ray Shri Pinkai Adak (*Alternate*)
Jawahar Lal Nehru Port Trust, Navi Mumbai	Manager (PPD) Deputy Manager (PPD) (*Alternate*)
Kolkata Port Trust, Kolkata	Shri A. K. Mehera Shri Amitabha Chattopadhyay (*Alternate*)
Larsen & Toubro Infrastructure Engineering Limited, Chennai	Shri N. Sunil Kumar Shri P. R. Rajesh (*Alternate*)
Military Engineer Services, Engineer-in-Chief's Branch, Integrated HQ of MoD (Army), New Delhi	Brig J. S. Ishar
Ministry of Shipping, New Delhi	Shri H. N. Aswath Shri Anil Pruthi (*Alternate*)
Mumbai Port Trust, Mumbai	Deputy Chief Engineer (D) Superintending Engineer (D) (*Alternate*)
National Centre for Coastal Research, Chennai	Dr M. V. Ramanamurthy Dr R. S. Kankara (*Alternate*)
National Institute of Ocean Technology, Chennai	Dr Basanta Kumar Jena Dr Vijaya Ravichandran (*Alternate*)

Organization	*Representative(s)*
Royal Haskoning DHV India, Noida	Shri Mohd. Aslam Bijapur Shri Satyanand Kunta (*Alternate*)
Simplex Infrastructure Ltd, Kolkata	Shri Atindra Narayan Basu
Visakhapatnam Port Trust, Visakhapatnam	Shri Amal Kumar Mehera Shri G. V. Satyanarayana (*Alternate*)
In personal capacity (*A-307, Casa Ansal Apts, 18 Bannerghatta Road, Bengaluru*)	Shri K. Vedagiri
Directorate General, BIS	Shri Sanjay Pant, Scientist 'F' and Head (Civil Engineering) [Representing Director General (*Ex-officio*)]

Member Secretary

Shri S. Arun Kumar
Scientist 'E' (Civil Engineering), BIS

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